CHAPTER 2

NON-LOAD RELATED DESIGN CONSIDERATIONS

2-1. **General.** Special circumstances may complicate the evaluation of bearing capacity such as earthquake and dynamic motion, soil subject to frost action, subsurface voids, effects of expansive and collapsible soil, earth reinforcement, heave in cuts and scour and seepage erosion. This chapter briefly describes these applications. Coping with soil movements and ground improvement methods are discussed in TM 5-818-7, EM 1110-1-1904 and EM 1110-2-3506.

2-2. **Earthquake and Dynamic Motion.** Cyclic or repeated motion caused by seismic forces or earthquakes, vibrating machinery, and other disturbances such as vehicular traffic, blasting and pile driving may cause pore pressures to increase in foundation soil. As a result, bearing capacity will be reduced from the decreased soil strength. The foundation soil can liquify when pore pressures equal or exceed the soil confining stress reducing effective stress to zero and causes gross differential settlement of structures and loss of bearing capacity. Structures supported by shallow foundations can tilt and exhibit large differential movement and structural damage. Deep foundations lose lateral support as a result of liquefaction and horizontal shear forces lead to buckling and failure. The potential for soil liquefaction and structural damage may be reduced by various soil improvement methods.

   a. **Corps of Engineer Method.** Methods of estimating bearing capacity of soil subject to dynamic action depend on methods of correcting for the change in soil shear strength caused by changes in pore pressure. Differential movements increase with increasing vibration and can cause substantial damage to structures. Department of the Navy (1983), "Soil Dynamics, Deep Stabilization, and Special Geotechnical Construction", describes evaluation of vibration induced settlement.

   b. **Cohesive Soil.** Dynamic forces on conservatively designed foundations with FS ≥ 3 will probably have little influence on performance of structures. Limited data indicate that strength reduction during cyclic loading will likely not exceed 20 percent in medium to stiff clays (Edinger 1989). However, vibration induced settlement should be estimated to be sure structural damages will not be significant.

   c. **Cohesionless Soil.** Dynamic forces may significantly reduce bearing capacity in sand. Foundations conservatively designed to support static and earthquake forces will likely fail only during severe earthquakes and only when liquefaction occurs (Edinger 1989). Potential for settlement large enough to adversely influence foundation performance is most likely in deep beds of loose dry sand or saturated sand subject to liquefaction. Displacements leading to structural damage can occur in more compact sands, even with relative densities approaching 90 percent, if vibrations are sufficient. The potential for liquefaction should be analyzed as described in EM 1110-1-1904.

2-3. **Frost Action.** Frost heave in certain soils in contact with water and subject to freezing temperatures or loss of strength of frozen soil upon thawing can alter bearing capacity over time. Frost heave at below freezing temperatures occurs from

formation of ice lenses in frost susceptible soil. As water freezes to increase the volume of the ice lense the pore pressure of the remaining unfrozen water decreases and tends to draw additional warmer water from deeper depths. Soil below the depth of frost action tends to become dryer and consolidate, while soil within the depth of frost action tends to be wetter and contain fissures. The base of foundations should be below the depth of frost action. Refer to TM 5-852-4 and Lobacz (1986).

    a. **Frost Susceptible Soils.** Soils most susceptible to frost action are low cohesion materials containing a high percentage of silt-sized particles. These soils have a network of pores and fissures that promote migration of water to the freezing front. Common frost susceptible soils include silts (ML, MH), silty sands (SM), and low plasticity clays (CL, CL-ML).

    b. **Depth of Frost Action.** The depth of frost action depends on the air temperature below freezing and duration, surface cover, soil thermal conductivity and permeability and soil water content. Refer to TM 5-852-4 for methodology to estimate the depth of frost action in the United States from air-freezing index values. TM 5-852-6 provides calculation methods for determining freeze and thaw depths in soils. Figure 2-1 provides approximate frost-depth contours in the United States.

    c. **Control of Frost Action.** Methods to reduce frost action are preferred if the depth and amount of frost heave is unpredictable.

    (1) Replace frost-susceptible soils with materials unaffected by frost such as clean medium to coarse sands and clean gravels, if these are readily available.

    (2) Pressure inject the soil with lime slurry or lime-flyash slurry to decrease the mass permeability.
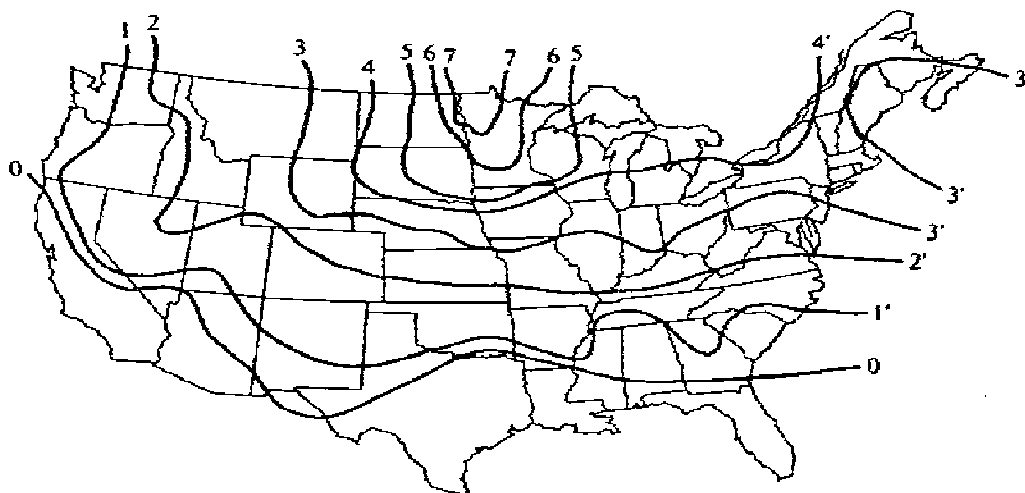

Figure 2-1. Approximate frost-depth contours in the United States.
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Analysis and Design", p. 305, 1988, by J. E. Bowles

(3) Restrict the groundwater supply by increased drainage and/or an impervious layer of asphalt, plastic or clay.

(4) Place a layer of thermal insulation such as foamed plastic or glass.

2-4. **Subsurface Voids.** A subsurface void influences and decreases bearing capacity when located within a critical depth D_c beneath the foundation. The critical depth is that depth below which the influence of pressure in the soil from the foundation is negligible. Evaluation of D_c is described in section 3-3b.

a. **Voids.** Voids located beneath strip foundations at depth ratios $D_c/B > 4$ cause little influence on bearing capacity for strip footings. B is the foundation width. The critical depth ratio for square footings is about 2.

b. **Bearing Capacity.** The bearing capacity of a strip footing underlain by a centrally located void at ratios $D_c/B < 4$ decreases with increasing load eccentricity similar to that for footings without voids, but the void reduces the effect of load eccentricity. Although voids may not influence bearing capacity initially, these voids can gradually migrate upward with time in karst regions.

c. **Complication of Calculation.** Load eccentricity and load inclination complicate calculation of bearing capacity when the void is close to the footing. Refer to Wang, Yoo and Hsieh (1987) for further information.

2-5. **Expansive and Collapsible Soils.** These soils change volume from changes in water content leading to total and differential foundation movements. Seasonal wetting and drying cycles have caused soil movements that often lead to excessive long-term deterioration of structures with substantial accumulative damage. These soils can have large strengths and bearing capacity when relatively dry.

a. **Expansive Soil.** Expansive soils consist of plastic clays and clay shales that often contain colloidal clay minerals such as the montmorillonites. They include marls, clayey siltstone and sandstone, and saprolites. Some of these soils, especially dry residual clayey soil, may heave under low applied pressure but collapse under higher pressure. Other soils may collapse initially but heave later on. Estimates of the potential heave of these soils are necessary for consideration in the foundation design.

(1) **Identification.** Degrees of expansive potential may be indicated as follows (Snethen, Johnson, and Patrick 1977):

Degree of Expansion	Liquid Limit, %	Plasticity Index, %	Natural Soil Suction, tsf
High	> 60	> 35	> 4.0
Marginal	50-60	25-35	1.5-4.0
Low	< 50	< 25	< 1.5

Soils with Liquid Limit (LL) < 35 and Plasticity Index (PI) < 12 have no potential for swell and need not be tested.

(2) **Potential Heave.** The potential heave of expansive soils should be determined from results of consolidometer tests, ASTM D 4546. These heave estimates should then be considered in determining preparation of foundation soils to reduce destructive differential movements and to provide a foundation of sufficient capacity to withstand or isolate the expected soil heave. Refer to TM 5-818-7 and EM 1110-1-1904 for further information on analysis and design of foundations on expansive soils.

b. **Collapsible Soil.** Collapsible soils will settle without any additional applied pressure when sufficient water becomes available to the soil. Water weakens or destroys bonding material between particles that can severely reduce the bearing capacity of the original soil. The collapse potential of these soils must be determined for consideration in the foundation design.

(1) **Identification.** Many collapsible soils are mudflow or windblown silt deposits of loess often found in arid or semiarid climates such as deserts, but dry climates are not necessary for collapsible soil. Typical collapsible soils are lightly colored, low in plasticity with LL < 45, PI < 25 and with relatively low densities between 65 and 105 lbs/ft^3 (60 to 40 percent porosity). Collapse rarely occurs in soil with porosity less than 40 percent. Refer to EM 1110-1-1904 for methods of identifying collapsible soil.

(2) **Potential Collapse.** The potential for collapse should be determined from results of a consolidometer test as described in EM 1110-1-1904. The soil may then be modified as needed using soil improvement methods to reduce or eliminate the potential for collapse.

2-6. **Soil Reinforcement.** Soil reinforcement allows new construction to be placed in soils that were originally less than satisfactory. The bearing capacity of weak or soft soil may be substantially increased by placing various forms of reinforcement in the soil such as metal ties, strips, or grids, geotextile fabrics, or granular materials.

a. **Earth Reinforcement.** Earth reinforcement consists of a bed of granular soil strengthened with horizontal layers of flat metal strips, ties, or grids of high tensile strength material that develop a good frictional bond with the soil. The bed of reinforced soil must intersect the expected slip paths of shear failure, Figure 1-3a. The increase in bearing capacity is a function of the tensile load developed in any tie, breaking strength and pullout friction resistance of each tie and the stiffness of the soil and reinforcement materials.

(1) An example calculation of the design of a reinforced slab is provided in Binquet and Lee (1975).

(2) Slope stability package UTEXAS2 (Edris 1987) may be used to perform an analysis of the bearing capacity of either the unreinforced or reinforced soil beneath a foundation. A small slope of about 1 degree must be used to allow the computer program to operate. The program will calculate the bearing capacity of the weakest slip path, Figure 1-3a, of infinite length (wall) footings, foundations, or embankments.

b. **Geotextile Horizontal Reinforcement.** High strength geotextile fabrics placed on the surface under the proper conditions allow construction of embankments and other structures on soft foundation soils that normally will not otherwise support pedestrian traffic, vehicles, or conventional construction equipment. Without adequate soil reinforcement, the embankment may fail during or after construction by shallow or deep-seated sliding wedge or circular arc-type failures or by excessive subsidence caused by soil creep, consolidation or bearing capacity shear failure. Fabrics can contribute toward a solution to these problems. Refer to TM 5-800-08 for further information on analysis and design of embankment slope stability, embankment sliding, embankment spreading, embankment rotational displacement, and longitudinal fabric strength reinforcement.

(1) **Control of Horizontal Spreading.** Excessive horizontal sliding, splitting, and spreading of embankments and foundation soils may occur from large lateral earth pressures caused by embankment soils. Fabric reinforcement between a soft foundation soil and embankment fill materials provides forces that resist the tendency to spread horizontally. Failure of fabric reinforced embankments may occur by slippage between the fabric and fill material, fabric tensile failure, or excessive fabric elongation. These failure modes may be prevented by specifyng fabrics of required soil-fabric friction, tensile strength, and tensile modulus.

(2) **Control of Rotational Failure.** Rotational slope and/or foundation failures are resisted by the use of fabrics with adequate tensile strength and embankment materials with adequate shear strength. Rotational failure occurs through the embankment, foundation layer, and the fabric. The tensile strength of the fabric must be sufficiently high to control the large unbalanced rotational moments. Computer program UTEXAS2 (Edris 1987) may be used to determine slope stability analysis with and without reinforcement to aid in the analysis and design of embankments on soft soil.

(3) **Control of Bearing Capacity Failure.** Soft foundations supporting embankments may fail in bearing capacity during or soon after construction before consolidation of the foundation soil can occur. When consolidation does occur, settlement will be similar for either fabric reinforced or unreinforced embankments. Settlement of fabric reinforced embankments will often be more uniform than non-reinforced embankments.

(a) Fabric reinforcement helps to hold the embankment together while the foundation strength increases through consolidation.

(b) Large movements or center sag of embankments may be caused by improper construction such as working in the center of the embankment before the fabric edges are covered with fill material to provide a berm and fabric anchorage. Fabric tensile strength will not be mobilized and benefit will not be gained from the fabric if the fabric is not anchored.

(c) A bearing failure and center sag may occur when fabrics with insufficient tensile strength and modulus are used, when steep embankments are constructed, or when edge anchorage of fabrics is insufficient to control embankment splitting. If

the bearing capacity of the foundation soil is exceeded, the fabric must elongate to develop the required fabric stress to support the embankment load. The foundation soil will deform until the foundation is capable of carrying the excessive stresses that are not carried in the fabric. Complete failure occurs if the fabric breaks.

     c. **Granular Column in Weak Soil.** A granular column supporting a shallow rectangular footing in loose sand or weak clay will increase the ultimate bearing capacity of the foundation.

     (1) The maximum bearing capacity of the improved foundation of a granular column supporting a rectangular foundation of identical cross-section is given approximately by (Das 1987)

$$q_u = K_p [\gamma_c D + 2(1 + \frac{B}{L}) C_u] \tag{2-1}$$

where

     K_p = Rankine passive pressure coefficient, $\dfrac{1 + \sin\phi_g}{1 - \sin\phi_g}$

     ϕ_g = friction angle of granular material, degrees
     γ_c = moist unit weight of weak clay, kip/ft^3
     D = depth of the rectangle foundation below ground surface, ft
     B = width of foundation, ft
     L = length of foundation, ft
     C_u = undrained shear strength of weak clay, ksf

Equation 2-1 is based on the assumption of a bulging failure of the granular column.

     (2) The minimum height of the granular column required to support the footing and to obtain the maximum increase in bearing capacity is 3B.

     (3) Refer to Bachus and Barksdale (1989) and Barksdale and Bachus (1983) for further details on analysis of bearing capacity of stone columns.

2-7. **Heaving Failure in Cuts.** Open excavations in deep deposits of soft clay may fail by heaving because the weight of clay beside the cut pushes the underlying clay up into the cut, Figure 2-2 (Terzaghi and Peck 1967). This results in a loss of ground at the ground surface. The bearing capacity of the clay at the bottom of the cut is $C_u N_c$. The bearing capacity factor N_c depends on the shape of the cut. N_c may be taken equal to that for a footing of the same B/W and D/B ratios as provided by the chart in Figure 2-3, where B is the excavation width, W is the excavation length, and D is the excavation depth below ground surface.

     a. **Factor of Safety.** FS against a heave failure is FS against a heave failure should be at least 1.5. FS resisting heave at the excavation bottom caused by seepage should be 1.5 to 2.0 (TM 5-818-5).

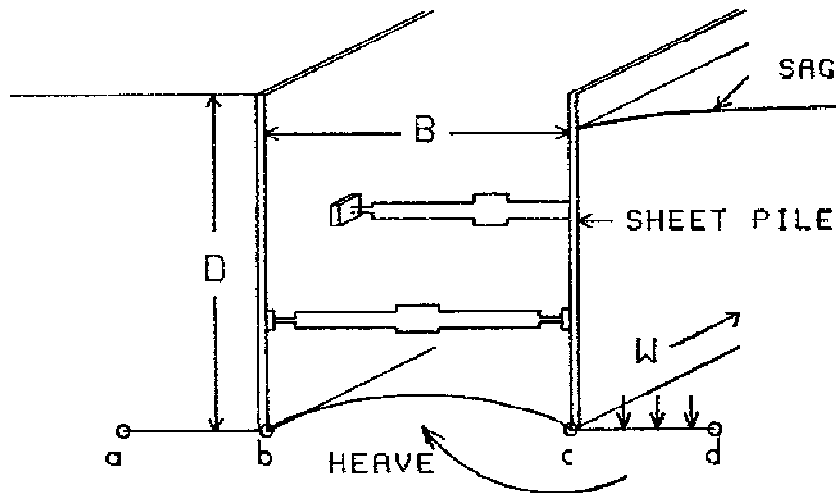$$FS = \frac{C_u N_c}{\gamma D} > 1.5 \tag{2-2}$$
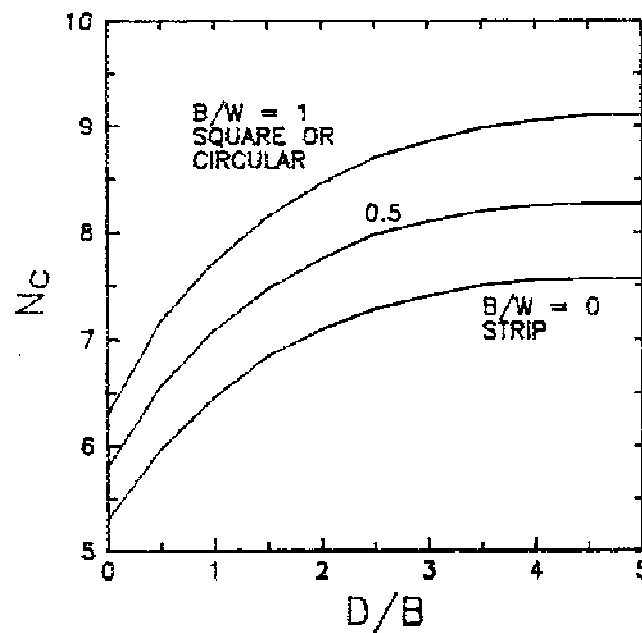
Figure 2-2. Heave failure in an excavation

Figure 2-3. Estimation of bearing capacity factor N_c for
heave in an excavation (Data from Terzaghi and Peck 1967)

    b. **Minimizing Heave Failure.** Extending continuous sheet pile beneath the
bottom of the excavation will reduce the tendency for heave.

    (1) Sheet pile, even if the clay depth is large, will reduce flow into the
excavation compared with pile and lagging support.

(2) Driving the sheet pile into a hard underlying stratum below the excavation greatly reduces the tendency for a heave failure.

2-8. **Soil Erosion and Seepage.** Erosion of soil around and under foundations and seepage can reduce bearing capacity and can cause foundation failure.

a. **Scour.** Foundations such as drilled shafts and piles constructed in flowing water will cause the flow to divert around the foundation. The velocity of flow will increase around the foundation and can cause the flow to separate from the foundation. A wake develops behind the foundation and turbulence can occur. Eddy currents contrary to the stream flow is the basic scour mechanism. The foundation must be constructed at a sufficient depth beneath the maximum scour depth to provide sufficient bearing capacity.

(1) **Scour Around Drilled Shafts or Piles in Seawater.** The scour depth may be estimated from empirical and experimental studies. Refer to Herbich, Schiller and Dunlap (1984) for further information.

(a) The maximum scour depth to wave height ratio is ≤ 0.2 for a medium to fine sand.

(b) The maximum depth of scour S_u as a function of Reynolds number R_e is (Herbich, Schiller and Dunlap 1984)

$$S_u = 0.00073 R_e^{0.619}$$

(2-3)

where S_u is in feet.

(2) **Scour Around Pipelines.** Currents near pipelines strong enough to cause scour will gradually erode away the soil causing the pipeline to lose support. The maximum scour hole depth may be estimated using methodology in Herbich, Schiller, and Dunlap (1984).

(3) **Mitigation of Scour.** Rock-fill or riprap probably provides the easiest and most economical scour protection.

b. **Seepage.** Considerable damage can occur to structures when hydrostatic uplift pressure beneath foundations and behind retaining walls becomes too large. The uplift pressure head is the height of the free water table when there is no seepage. If seepage occurs, flow nets may be used to estimate uplift pressure. Uplift pressures are subtracted from total soil pressure to evaluate effective stresses. Effective stresses should be used in all bearing capacity calculations.

(1) Displacement piles penetrating into a confined hydrostatic head will be subject to uplift and may raise the piles from their end bearing.

(2) Seepage around piles can reduce skin friction. Skin friction resistance can become less than the hydrostatic uplift pressure and can substantially reduce bearing capacity. Redriving piles or performing load tests after a waiting period following construction can determine if bearing capacity is sufficient.